GEOTECHNICAL

GEODESIGN INC

Geotechnical • Environmental • Geological

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Engineering Lab Building

Fourth Avenue

Portland, Oregon

GDI Project: PSU-4-01

For

Portland State University

GEODESIGN INC
Geotechnical • Environmental • Geological

April 19, 2002

Portland State University
617 SW Montgomery
Portland, OR 97207-0751

Attention: Mr. Richard B. Piekenbrock

Report of Geotechnical Engineering Services
Engineering Lab Building
Fourth Avenue
Portland, Oregon
GDI Project: PSU-4-01

GeoDesign is pleased to submit our report of geotechnical engineering services for Portland State University's proposed Engineering Lab Building. The site is located adjacent to (south) the City Development Center located at 1900 SW 4th Avenue in Portland, Oregon. Our services for this project were conducted in accordance with our proposal dated January 29, 2002.

We appreciate the opportunity to be of continued service to you. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

George Saunders, P.E.
Principal

cc: Mr. Peter W. Van der Meulen, Zimmer Gunsul Frasca Partnership (four copies)
Mr. Ed Quesenberry, kpff Consulting Engineers (one copy)

CJP:GPS:kt
Attachments
Four copies submitted
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INTRODUCTION

This report presents the results of GeoDesign's geotechnical engineering evaluation of Portland State University's (PSU) proposed Engineering Laboratory Building. The site is located adjacent to the City Development Center located at 1900 SW 4th Avenue in Portland, Oregon. The site location relative to surrounding physical features is shown in Figure 1.

Mr. Richard Piekenbrock provided us with copies of our June 25, 1998 geotechnical report for the adjacent City Development Center, architectural drawings of the proposed development prepared by Zimmer Gunsul, Frasca Partnership (ZGF), and Sheets S-1 through S-4 of the structural drawings for the existing parking structure at the site. ZGF provided us with schematic design drawings for the project. Additional project information was obtained through telephone conversations with Mr. Peter Vandermeulen of ZGF and Mr. Ed Quesenberry of kpff Consulting Engineers (kpff).

The development plans are to construct a five-story building over the existing three-level, below-grade parking structure at the site. Construction of the structure will be staged with the removal of the existing landscaping overlying the roof grade of the parking structure. Based on information supplied by kpff, we understand that the new column loads will vary between 500 and 2,800 kips and with few exceptions, the new columns will bear on the existing columns and footings of the parking structure.

Based on schematic design drawings provided by ZGF, new spread footings will be constructed at the southeast corner of the parking structure and at the elevator pit. Four spread footings will be constructed at the proposed elevator pit to temporarily relieve loads from adjacent footings during excavation of the pit and construction of the embedded walls. Shoring will be required for lateral support during construction. In most cases, footings will be constructed immediately adjacent to the existing footings.

BACKGROUND

Our June 25, 1998 report for the City Development Center was prepared based on a review of Shannon and Wilson, Inc.'s (SWI) May 2, 1973 report for the site. The SWI report included the results of borings they completed as well as the results from prior explorations completed by others in the area of the proposed new structure.

At the time of SWI's report, plans for the site were to construct a building in three phases, with the first phase the current three-level parking structure and the final phase resulting in a 14-story tower constructed in the area currently occupied by the City Development Center. The report provided recommendations for mat foundations for the 14-story tower, and spread foundations for the smaller structures. Based on as-built structural drawings, the footings for the existing parking structure were sized for an allowable bearing pressure of 4.5 kips per square foot (ksf).

PURPOSE AND SCOPE

The purpose of our services was to evaluate the existing information and explore the subsurface conditions at the site to provide the basis for geotechnical recommendations for site development and foundation and design. Based on our review of the available information and our experience in the area, we proposed to complete an additional exploration to at the southwest corner of the parking structure. Our specific scope of work included the following:

- Review the existing information, including our analysis associated with our June 25, 1998 report and SWI's May 2, 1973 report.
- Coordinate and manage the field investigation including utility locates, site access authorizations, access preparation, traffic control, and scheduling of contractors and of GeoDesign's staff.
- Complete one cone penetrometer (CPT) probe advanced to an approximate depth of 100 feet, or the depth of the underlying gravel formation. Complete shear wave velocity tests at 1-meter intervals.
- Provide our evaluation and recommendations for construction of the new building on the existing foundation of the parking structure.
- Provide geotechnical engineering recommendations for design and construction of shallow spread foundations, including allowable design bearing pressure, and minimum footing depth and width.
- Recommend design criteria for retaining walls, including static and dynamic lateral earth pressures, backfill, compaction, and drainage.
- Provide recommendations for construction of asphalt pavements for on-site access roads and parking areas, including subbase, base course, and asphalt paving thickness.
- Provide recommendations for the management of identified groundwater conditions that may affect the performance of structures or pavement.
- Evaluate liquefaction potential of site soils, including estimates of liquefaction-induced settlement and lateral spread. If necessary, provide a discussion of methods to improve the liquefaction resistance of site soils.
- Complete a seismic hazard evaluation of the site in accordance with Uniform Building Code requirements.
- Provide a written report summarizing our recommendations.

SITE CONDITONS

SURFACE CONDITONS

The site is bordered by SW 4th Avenue on the west and the 1900 SW 4th City Development Center building on the north. A below-grade parking structure occupies the site. The top of the parking structure is currently landscaped with grass areas and trees. Based on the information provided to us, the ground surface elevation in the area is between approximately 135 and 147 feet and the elevation of the lower parking level is approximately 106 feet.

SUBSURFACE CONDITIONS

The May 2, 1973 geotechnical engineering report prepared by SWI provides the results of eight borings completed by Dames & Moore and four borings by SWI. A copy of the site plan, boring logs, and profiles from the SWI report is provided in Appendix A.

The subsurface conditions below the foundation elevation of the existing building consist of approximately 60 feet of medium dense to very dense, fine, silty sand underlain by very dense gravels from the Troutdale Formation. Results from triaxial tests on the silty sand indicate an effective angle of internal friction of 39 to 40 degrees. Results from consolidation testing indicate a moderate degree of over consolidation and relatively low to moderate compressibility.

We advanced one CPT probe to a depth of 100 feet adjacent to the southwest corner of the parking structure at the approximate location shown in Figure 2. Due to access restrictions, the probe was located in the eastern lane of SW 4th Avenue just south of the existing entrance to the parking structure. In general, the subsurface conditions observed in the CPT exploration agree with the conditions observed in the previous explorations. The gravel formation was not encountered in the CPT exploration to the depth of 100 feet. CPT data is included in Appendix B.

Based on information provided in the SWI report, groundwater was encountered in one exploration at greater than 110 feet below ground surface (bgs). A monitoring well was installed in that exploration and additional monitoring indicated that the water observed may have been perched and the actual groundwater level was deeper than the boring. Groundwater was apparently not encountered in Dames & Moore's explorations.

Our CPT exploration confirmed deep groundwater at the site. Pore pressure measurements indicate groundwater at approximately 93 feet bgs (elevation 54 feet). However, this groundwater may be the same perched water encountered the SWI's boring. According to the SWI report, local deep wells in the vicinity of the site suggest that the true groundwater level is at approximately elevation 10 feet. Based on the available information, groundwater should not affect design and construction at this site.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on our review of the available information and the results of our exploration and analyses, it is our opinion the proposed structure with the column loads previously discussed can be supported on the existing foundations of the parking structure. The results of our seismic hazard investigation indicate that there is a low seismic hazards for landslides, fault rupture, tsunamis, liquefaction, and amplification at the site. The following paragraphs present specific geotechnical recommendations.

SITE PREPARATION AND CONSTRUCTION CONSIDERATIONS

Development plans include removal of the existing landscape materials overlying the parking structure. However, landscape or fill soils will likely remain around the perimeter of the new structure. These areas may require granular working blankets and/or haul roads during construction and significant over-excavation may be necessary for support of new pavements, if planned. The following paragraphs provide our standard recommendations for evaluation of subgrade, erosion control, and construction of working blankets and haul roads.

We recommend proofrolling the subgrade in new sidewalk and pavement areas using a fully loaded dump truck or similar-size, rubber-tire construction equipment to identify areas of excessive yielding. A member of our geotechnical staff should observe the proof roll to evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted materials recommended for structural fill. Areas of subgrade that appear to be too wet and soft to support proofrolling equipment should be evaluated by probing with a steel rod, rather than by proof rolling. Wet soil that has been disturbed during site preparation activities, or soft or loose zones identified during probing, should be removed and replaced with granular structural fill.

Silt fences, hay bales, and granular haul roads should be used as required to reduce sediment transport during construction to acceptable levels. Measures to reduce erosion should be implemented in accordance with Oregon Administrative Rules 340-41-006 and 340-41-455, and the City of Portland and Multnomah County regulations regarding erosion control

Granular haul roads and staging areas may be necessary for support of construction traffic in existing landscaped areas or in areas where the existing pavement has been removed. A 12-inch-thick layer of imported granular material generally should be sufficient for light staging areas, but is generally not expected to be adequate to support heavy equipment or truck traffic. Haul roads and areas with repeated heavy construction traffic should be constructed with a minimum thickness of 18 inches of imported granular material. The imported granular material should consist of crushed rock that is well graded and has less than 7 percent by weight passing the U.S. Standard No. 200 Sieve. It may be possible to use recycled asphalt and baserock in haul roads and staging areas if the material meets the aforementioned gradations. A geotextile should be placed in the haul roads below the granular material and should have a minimum Mullen burst strength of 250 pounds per square inch (psi) for puncture resistance and an apparent opening size (AOS) between an U.S. Standard No. 70 and No. 100 Sieve.

STRUCTURAL FILL

Granular material used as structural fill should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine, contains no organic matter or other deleterious materials, has a maximum particle size of 3 inches, and has less than 5 percent passing the U.S. Standard No. 200 Sieve. Imported granular material should be moisture-conditioned to the approximate optimum moisture content, placed in 12-inch-thick lifts, and compacted to not less than 95 percent of maximum dry density as determined by American Society for Testing and Materials (ASTM) D 1557.

SHALLOW FOUNDATIONS

Existing Spread Footings

We understand that a significant portion of the dead load on the existing footings is from the surface landscape layer. Based on our discussions with kpff, the loads associated with the proposed structure following removal of the landscape layer corresponds to an increase in bearing pressure of up to 1,500 pounds per square foot (psf) for the existing footings of the parking structure. The structural notes on the plans for the parking structure indicate that the footings were originally designed for a bearing pressure of 4.5 ksf. Our settlement analysis is based on the boring logs and consolidation test results presented in the SWI report and indicates that the total settlement associated with a 1,500 psf increase in bearing pressure will be less than approximately 1 inch. Differential settlements should be less than approximately ½ inch. Settlement should occur rapidly as the additional loads are applied.

New Spread Footings

We recommend that new spread footings bear on undisturbed medium dense to very dense, sandy silt and silty sand, or structural fill that is properly installed during construction and underlain by undisturbed native materials. A minimum 6 inches of crushed rock compacted as structural fill should be placed in the base of the footing excavations to protect the native materials from disturbance during construction.

Spread footings should have a minimum width of 24 inches, with the base of the footings founded at least 12 and 18 inches below the lowest adjacent grade for interior and exterior footings, respectively. Continuous wall footings should have a minimum width of 18 inches, and be founded a minimum of 18 inches below the lowest adjacent grade.

New footings founded as recommended should be proportioned for a maximum allowable soil bearing pressure of 4,500 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and may be increased by a third when considering earthquake or wind loads. The weight of the footing and overlying backfill can be ignored in calculating footing loads.

For a 4,500-psf design bearing pressure, total settlement of new footings is anticipated to be less than approximately 1 inch for the anticipated building loads. Differential settlements should not exceed ½ inch.

Lateral Capacity

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by structural fill or for footings constructed in direct contact with the undisturbed native soil is 350 pounds per cubic foot (pcf). Typically, the movement required to develop the available passive resistance may be relatively large. Therefore, we recommend using a reduced passive pressure of 300 pcf. Adjacent floor slabs or pavements should not be considered when calculating passive resistance.

A coefficient of friction equal to 0.35 may be used when calculating resistance to sliding.

SHORING AND EMBEDDED BUILDING WALLS

Design drawings provided by ZGF show that temporary shoring and embedded walls adjacent to existing footings are required for construction of the elevator pit. Our design recommendations for the shoring and embedded walls assume a maximum bearing pressure of the footings is less than 6,000 psf.

We recommend that the shoring and embedded walls be designed for an equivalent fluid pressure of 60 pcf **plus** a lateral pressure of 3,000 psf due to the influence of the adjacent footing. Passive resistance at the toe of the shoring should be calculated using an equivalent fluid pressure of 300 pcf. Passive resistance should be neglected above 1 foot below the base of the excavation.

RETAINING STRUCTURES

The following design recommendations for conventional retaining structures are based on the assumptions that: (1) the walls consist of conventional cantilevered retaining walls, (2) the walls are less than 10 feet in height, (3) the backfill is level and drained and consists of imported granular materials, and (4) no surcharges are imposed behind the wall. Reevaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

For walls not restrained from rotation, we recommend using an equivalent fluid pressure of 36 pcf for design. We recommend using an equivalent fluid pressure of 55 pcf for design of walls restrained from rotation. Footings for the retaining walls should be designed in accordance with the recommendations given for shallow spread footings.

As stated above, our recommendations are based on the assumption of drained conditions. Drains that consist of a 6- to 8-inch-diameter perforated drainpipe should be installed behind all retaining structures. The pipe should be embedded in a minimum 3-foot-wide zone of drain rock and sloped to drain (minimum slope of ½ percent) toward a suitable discharge. The drain rock should be wrapped in a geotextile fabric. Backfill material placed behind the wall and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of the well-graded gravel, with not more than 5 percent by weight passing the U.S. Standard No. 200 Sieve. Alternatively, fine-grained soils or the on-site silty sand can be used as backfill material provided a minimum 3-foot-wide column of drain rock wrapped in a geotextile is placed against the wall. The rock column should extend from the foundation drains to within approximately 1 foot of the ground surface.

The geotextile should have an AOS between the U.S. Standard No. 70 and No. 100 Sieve and a water permittivity greater than 1.5 sec^{-1}. The drain rock should be uniformly graded, have a maximum particle size of 3 inches, and have less than 2 percent passing the U.S. Standard No. 200 Sieve (washed analysis).

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as jumping jack or vibratory plate compactors). If flat work (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D 1557. Settlements of up to 1 percent of the wall height commonly occur immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flat work adjacent to retaining walls be postponed at least four weeks after construction, unless survey data indicates that settlement is complete prior to that time.

PAVEMENT RECOMMENDATIONS

General

The pavement subgrade should be prepared in accordance with the previously described site preparation, construction considerations, and structural fill recommendations. Our pavement recommendations assume that paving will be conducted in an extended period of dry weather. Construction during wet conditions could require an increased thickness of aggregate base.

We do not have specific information on the frequency and type of vehicles that will use the area; however, we have assumed that traffic conditions will consist of fewer than 10 trucks per day. We should reevaluate these thicknesses if traffic exceeds this assumption. We used a subgrade resilient modulus of 6,000 psi in our analyses.

We recommend a section consisting of 3.0 inches of asphalt concrete (AC) over 9.0 inches of aggregate base in areas where truck traffic is expected. If parking areas are limited to passenger automobiles only, the pavement section can be reduced to 2.5 inches of AC over 6.0 inches of crushed rock.

The AC pavement should conform to Section 00745 for standard- and heavy-duty asphalt pavements of the Supplemental Standard Specifications for Highway Construction, Oregon Department of Transportation, 1996 Edition. The crushed rock base should conform to Section 02630 of Standard Specifications for Highway Construction, Oregon Department of Transportation (ODOT), 1996 Edition and have less than 5 percent passing the U.S. Standard No. 200 Sieve. Crushed rock base should be placed in a single lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

SEISMIC DESIGN

Uniform Building Code Design Criteria

We recommend that the building be designed using the applicable provisions of the State of Oregon Structural Specialty Code for Zone 3. Based on shear wave velocities measured in our CPT exploration, site conditions correspond to a S_D site soil profile type, resulting in Uniform Building Code (UBC) 1997 C_a and C_v values of 0.36 and 0.54, respectively. Specific seismic design criteria and a seismic hazard evaluation are presented in Appendix C.

Liquefaction

Liquefaction settlement is the result of seismically induced densification and subsequent ground settlement of loose/soft, saturated soils. DOGAMI's mapping of liquefaction hazards indicates the site is in an area with a low risk of liquefaction. Our liquefaction analysis based on data from our CPT exploration indicates that the medium dense to very dense sand and dense to very dense gravel underlying the existing structure are not liquefiable. Due to the absence of liquefiable soils and the topography of the site, the risk of lateral spreading is low. Additional seismic information is presented in Appendix C and CPT data is presented in Appendix B.

OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including proofrolling of the pavement area subgrades and repair of soft areas, performing laboratory and compaction and field moisture-density tests of fill materials, observing final proofrolling of the pavement subgrade and base rock, and observing footing subgrade preparation.

LIMITATIONS

We have prepared this report for use by Portland State University and their design and construction team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Our scope has included one exploration and review of several prior explorations completed by others. The explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, reevaluation will be necessary.

The site development plans and design details were not final at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the buildings, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty, express or implied, should be understood.

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Christopher Palmer, E.I.T.
Geotechnical Staff III

George Saunders, P.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
18,028
OREGON
JULY 29, 1980
GEORGE P. SAUNDERS

6/30/2002